Chapter 3

Experimentation
3.1 IDENTIFICATION OF SOIL

The soil type used in the investigation was identified and the engineering properties were determined in the laboratory by performing various tests as per following.

3.1.1 Grain Size Analysis:

The sieving was performed by arranging the various sieves one over the other in the order of their mesh opening – the largest aperture sieve being kept at the top and the smallest aperture sieve at the bottom. A receiver was kept at the bottom and a cover is kept at the top of the whole assembly. The soil sample was put on the top sieve, and the whole assembly was fitted on a sieve shaking machine. The shaking was done for ten minutes. The portion of the soil sample retained on each sieve was weighed. The percentage soil retained on each sieve was calculated on the basis of the total mass of soil sample taken and from the results, percentage passing through each sieve was calculated. Table- 3.1 shows the observation and calculation. The graph was then plotted for percentage passing and grain size. The values of $D_{10}$, $D_{30}$, $D_{60}$ were determined from the graph. From that the Coefficient of uniformity ($Cu$) & Coefficient of curvature ($Cc$) were calculated.

Where, $Cu = \frac{D_{60}}{D_{10}}$ & $Cc = \frac{(D_{30})^2}{(D_{60} * D_{10})}$.

Table- 3.1 Sieve analysis of research sand

<table>
<thead>
<tr>
<th>Sieve size(mm)</th>
<th>Particle size(mm)</th>
<th>Mass retained (gm)</th>
<th>% Retained</th>
<th>cumulative % Retained</th>
<th>cumulative % finer(N)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.75 mm</td>
<td>4.75</td>
<td>0</td>
<td>-</td>
<td>-</td>
<td>100</td>
</tr>
<tr>
<td>2.36 mm</td>
<td>2.36</td>
<td>1.5</td>
<td>0.15</td>
<td>0.15</td>
<td>99.85</td>
</tr>
<tr>
<td>1.18 mm</td>
<td>1.18</td>
<td>334</td>
<td>33.4</td>
<td>33.55</td>
<td>66.45</td>
</tr>
<tr>
<td>600 μ</td>
<td>0.6</td>
<td>359</td>
<td>35.9</td>
<td>69.45</td>
<td>30.55</td>
</tr>
<tr>
<td>300 μ</td>
<td>0.3</td>
<td>272</td>
<td>27.2</td>
<td>96.65</td>
<td>3.35</td>
</tr>
<tr>
<td>150 μ</td>
<td>0.15</td>
<td>27.5</td>
<td>2.75</td>
<td>99.4</td>
<td>0.6</td>
</tr>
<tr>
<td>75 μ</td>
<td>0.075</td>
<td>6</td>
<td>0.6</td>
<td>100</td>
<td>0</td>
</tr>
</tbody>
</table>

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Particle size distribution curve

From Graph:

\[ D_{10} = 0.36 \]

\[ D_{30} = 0.6 \]

\[ D_{60} = 1.1 \]

\[ C_u = 3.05 \]

\[ C_c = 0.91 \]

Type of soil: SP (Poorly graded Sand)

3.1.2 Specific Gravity Test:

A clean and dry pycnometer was taken for the test. The empty weight of pycnometer (W1) was done and about 200 to 400g of oven-dried sand was taken in to the pycnometer. Weight of the pycnometer plus the dry soil (W2) was taken. Then distilled water was added in the pycnometer to its half the height and mixed it thoroughly with glass rod. Added more water & stirred it well. Pycnometer was
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filled with water up to its top of the conical cap. After drying the pycnometer from outside weight (W3) was recorded and emptied the pycnometer, cleaned it thoroughly and filled it with distilled water up to the hole of the conical cap and recorded the weight (W4). The specific gravity, then calculated by using the following equation.

\[
G = \frac{W_2 - W_1}{(W_4 - W_1) - (W_3 - W_2)}
\]

<table>
<thead>
<tr>
<th>Sr.No.</th>
<th>Description</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Wt. of empty pycnometer (W1)</td>
<td>449gm</td>
</tr>
<tr>
<td>2</td>
<td>Wt. of empty pycnometer + Dry soil (W2)</td>
<td>866gm</td>
</tr>
<tr>
<td>3</td>
<td>Wt. of empty pycnometer + Dry soil + Water (W3)</td>
<td>1679gm</td>
</tr>
<tr>
<td>4</td>
<td>Wt. of empty pycnometer + Water</td>
<td>1423gm</td>
</tr>
</tbody>
</table>

Specific Gravity = 2.59

G = 2.59

3.1.3 Relative Density Test:

Minimum dry density (\(\gamma_{\text{dmin.}}\)) = 15.3 kN/m\(^3\)

Maximum dry density (\(\gamma_{\text{dmax}}\)) = 18.03 kN/m\(^3\)

Field density maintained during tests \(\gamma_d = 16.7\) kN/m\(^3\).

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Relative Density, \( R_d = \frac{(\gamma_d - \gamma_{d_{\text{min}}})}{(\gamma_{d_{\text{max}}} - \gamma_{d_{\text{min}}})} \times \frac{\gamma_{d_{\text{max}}}}{\gamma_d} \times 100 \)

- \( R_d = 55.37\% \)

So, the representative sample comes under the **Medium dense** category.

### 3.1.4 Direct shear Test:

The shear parameters of the representative soil sample with the same density (16.7 kN/m\(^3\)) as used in the model loading tests was determined using simple direct shear box test.

The shear parameters determined after plotting the graph for shear stress vs normal stress are as under:

- Cohesion parameter \((C) = 0.0\)
- Angle of internal friction \(\varphi = 36\degree\)

The characteristics of sand used in the study is given in table- 3.2

**Table 3.2 Properties of research sand**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific gravity (G)</td>
<td>2.6</td>
</tr>
<tr>
<td>Maximum density ((\gamma_{-\text{max}}))</td>
<td>18.03 kN/m(^3)</td>
</tr>
<tr>
<td>Minimum density ((\gamma_{-\text{min}}))</td>
<td>15.30 kN/m(^3)</td>
</tr>
<tr>
<td>Compacted density ((\gamma))</td>
<td>16.7 kN/m(^3)</td>
</tr>
<tr>
<td>Coefficient of curvature (Cc)</td>
<td>0.91</td>
</tr>
<tr>
<td>Coefficient of uniformity (Cu)</td>
<td>3.05</td>
</tr>
<tr>
<td>Cohesion parameter ((C))</td>
<td>0.0 kN/m(^2)</td>
</tr>
<tr>
<td>Angle of internal friction (\varphi)</td>
<td>36\degree</td>
</tr>
</tbody>
</table>
3.2 **Materials used for Reinforcement:**

Total four types of reinforcing materials are used in the investigation as follows

- Netlon geogrid-CE121
- Netlon geogrid-CE131.
- Geojute
- Three dimensional geocells made up of Netlon geogrid-CE121.

The characteristics of geogrids are shown in Table 3.3 and the characteristics of geojute and geocells is shown as follows.

**Table 3.3 Characteristics of geogrids**

<table>
<thead>
<tr>
<th>Geogrid-(Netlon)</th>
<th>CE121</th>
<th>CE131</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness at node</td>
<td>2.75mm</td>
<td>5.5mm</td>
</tr>
<tr>
<td>Thickness of rib</td>
<td>2.2mm</td>
<td>2.5mm</td>
</tr>
<tr>
<td>Aperture size</td>
<td>7.5*7.5mm</td>
<td>28*28mm</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>7.68 N/mm</td>
<td>5.8 N/mm</td>
</tr>
<tr>
<td>Yield strength</td>
<td>6.8 N/mm</td>
<td>5.2 N/mm</td>
</tr>
</tbody>
</table>

**Characteristics of Geo-jute**

Thickness \((t) = 3.5\text{mm}\)

Tensile Strength = 13.2 N/mm

Natural Material

**Characteristics of geocells**

\(D_g\) – diameter of geocell

\(H_g\) – height of geocell

Fig.3.1 to Fig.3.4 show the photographs of the reinforcing materials used in the investigation.
Fig. 3.1 Netlon geogrid CE121
Fig. 3.2 Geo-jute
Fig. 3.3 Netlon geogrid CE131
Fig. 3.4 Geocells of various sizes.
3.3 TEST PROGRAM:

After determining the physical and engineering properties of soil and the reinforcements the experimental work was carried out in three series:

3.3.1 FIRST SERIES OF EXPERIMENTS

The first series involves monotonic and cyclic model tests performed under ring footing resting on sand with reinforcing materials as Geogrid CE-121 and geo-jute. The following parameters were studied for both the planar reinforcing materials

- Depth of top layer of reinforcement below the model footing, u,
- Number of reinforcing layers, N,
- Vertical spacing between adjacent layers, z,
- Size of the reinforcing material, b.

The model footing used in this series was a ring having external diameter \((d) = 130\text{mm}\) and internal diameter \((h) = 50\text{mm}\). The parameters \(u\) and \(z\) as defined earlier were kept constant which is equal to \(0.25d\).

Preparation of sand beds: The sand bed was prepared in a square steel tank of size \(0.8\text{m} \times 0.8\text{m} \times 0.8\text{m}\) by placing the sand in \(100\text{mm}\) lifts and compacting each lift to the desired density of \(16.7 \text{kN/m}^3\). The reinforcing layers were placed in horizontal position at a required depth below the model footing and a thin layer of sand followed by more sand was carefully placed over the reinforcing layer & then compacted before the model footing was placed. Model footing was then placed so that the centre of the plate coincides with the centre of the reaction girder, with the help of a plumb and bob over the level bed, a proving ring of \(5t\) capacity was placed on the model footing and two dial gauges were fixed on opposite sides of the proving ring to measure the settlement of the footing. The experimental set-up is shown in Fig.3.5.
Fig. 3.5 Experimental set-up
Plate- load test under monotonic loading: After the experimental set-up was ready, the load was applied in cumulative equal increments up to one fifth of the estimated ultimate bearing capacity. The load was applied without impact, fluctuation or eccentricity and the load was measured over the proving ring. Settlements were observed for each increment of the load after the rate of settlement became negligible. The next increment of load was then applied and observations repeated. The test was continued till a settlement of 10% of the footing size was achieved or the soil bed failed under shear.

For all the monotonic tests, load vs settlement curves were plotted and the ultimate bearing pressure $q_o$ for the un-reinforced sand has been compared with the ultimate bearing pressure of reinforced sand $q$, and the $BCR = \frac{q}{q_o}$ has been computed for all the tests. Another dimensionless parameter named settlement ratio factor, $SRF$ has been used to represent the reduction in settlement for reinforced condition. $SRF = \frac{S}{S_0}$, where $S$ = settlement for reinforced condition at a bearing pressure corresponding to the ultimate bearing pressure of the un-reinforced sand and $S_0$ = settlement corresponding to ultimate bearing pressure of un-reinforced condition. The relationship between pressure and settlement obtained from the tests, have been studied by plotting curves for all the tests of the series conducted. For all test conditions $E_i$ = initial tangent modulus were determined from the curves, and also the modular ratio $m = \frac{E_i}{E_{i0}}$. Load-settlement readings of all the tests of series -1, with geo-grid (CE121) and geo-jute reinforced sand beds are shown in Table 4.1.1 to 4.1.25 and load settlement curves are shown in Fig. 4.1.1 to 4.1.6.

Cyclic plate load test: Cyclic plate load tests were carried out as per IS 5249:1992. After the setup has been arranged, the initial readings of the dial gauges were noted and the first increment of static load was applied to the footing. The load was maintained constant throughout for a period till no further settlement occurred or the rate of settlement became negligible. The final readings of dial gauges were then recorded. The entire load was then removed quickly but gradually and the
footing was allowed to rebound. When no further rebound occurred or the rate of rebound became negligible, the readings of the dial gauges were again noted. The load was then increased gradually till its magnitude acquired a value equal to proposed next higher stage of loading, which was maintained constant & the final dial gauge readings were noted as mentioned earlier. The entire load was then reduced to zero and final dial gauge readings were recorded when the rate of rebound became negligible. The cycles of loading & unloading and reloading continued till the estimated ultimate load had been reached, the final values of dial gauge readings had been noted each time. The cyclic plate load tests were conducted on prepared beds to study the influence of above parameters on damping capacities of sand bed. Load – settlement curves for each test set has been plotted. A typical load – settlement curve is shown in Fig. 3.6. The curves were also plotted for W* vs S* for each set of loading and unloading hence hysteresis loops were obtained for the test series. The indicative of damping capacities of reinforced sand and unreinforced sand are determined by calculating area of one hysteresis loop and area under W* vs S* with the help of planimeter. The test results of the comparison between damping capacity of unreinforced sand bed with that of reinforced sand beds are shown in the next chapter.

Load (Q)

<table>
<thead>
<tr>
<th>Sp</th>
<th>Se</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Fig. 3.6 Load- settlement curve for cyclic plate load test

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The indicative of damping capacity obtained by the ratio $\Delta E/E_0$.

Where, $\Delta E = \text{Area of hysteresis loop}$,

$E_0 = \text{Area under } W^* \text{ vs } S^*$,

$W^* = \text{Load parameter } [p + (\gamma \cdot d)]$,

$S^* = \text{settlement parameter } (S + d)$,

$d = \text{outer diameter of ring footing},$

$p = \text{load intensity on model footing},$

$\gamma = \text{density of sand},$

The sequential loading and unloading adopted for all tests made it possible to separate the recoverable component ($S_e$) and non-recoverable component ($S_p$) of the total settlement of the footing for different load levels. The coefficient of elastic uniform compression $Cu$, the coefficient of elastic shear $C_t$, the coefficient of elastic non-uniform shear $C_x$, and the coefficient of elastic non-uniform compression $C_p$ are then determined by the relations given below as per IS 5249: 1992.

$$Cu = \frac{P}{S_e} \text{ kN/m}^3,$$

$Cu$ can be calculated as shown in Fig. 3.7

![Fig. 3.7 Determination of the coefficient of elastic uniform compression](image)

$Cu$
Where, $p =$ corresponding load intensity in kN/m$^2$ and 
$S_e =$ Elastic rebound settlement corresponding to $p$ in m.
$C_u = 1.5$ to $2 \ C_t$, 
$C_\varphi = 3.46 \ C_t$, 
$C_\psi = 1.5 \ C_t$.

### 3.3.2 SECOND SERIES OF EXPERIMENTS:

In the second series of experimentation, an attempt was made to study the effect of annular ratio $(h/d)$ of the ring footing on reinforced sand bed which involves model plate load tests under monotonic vertical loading on geogrid reinforced sand bed. The model footings used in this series were as follows:

- Ring having external diameter, $d=130$mm and internal diameter, $h=40$mm
- Ring having external diameter, $d=130$mm and internal diameter, $h=50$mm
- Ring having external diameter, $d=130$mm and internal diameter, $h=60$mm
- Solid circular footing with external diameter $d=130$mm i.e. $h=0$.

In all the tests of this series the size of reinforcing elements were kept same and that is equal to three times the external diameter of footing. The first layer of reinforcement was kept at a dept $u = 0.25d$ and the spacing of the reinforcing layers $Z = 0.25d$ for all the tests under reinforcing condition. In this series plate load test were performed only under monotonic loading.

### 3.3.3 THIRD SERIES OF EXPERIMENTS:

Laboratory plate load tests were performed under monotonic and cyclic loading conditions on the sand bed reinforced with geocells of various sizes in the third series. The parameters varied in this series are as under

- Depth of top of the geocell layer below model footing, $Z$, 

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- Diameter of geocell, $D_g$,
- Height of geocell, $H_g$ and
- Number of geocell, $N$.

The model footing used in this series is a ring having external diameter ($d$) = 130mm and internal diameter ($h$) = 50mm. Plate load tests under monotonic and cyclic loading were performed as described in the previous section 3.3.1. The details of the plate – load tests performed with geocell reinforcement are as under.

- Monotonic plate load tests on Geocell (made from geogrid CE-121) reinforced sand bed having $H_g=80$ mm ($0.61d$), $D_g=35$ mm ($0.27d$) with $N=1$, $N=2$, $N=3$, geocells with $Z=0.25d$, $Z=0.5d$, $Z=0.75d$ & Cyclic plate load tests with same size of geocells and depth of placement with $N=1$ to $N=4$.

- Monotonic plate load tests on Geocell (made from geogrid CE-121) reinforced sand bed having $H_g=80$ mm ($0.61d$), $D_g=60$ mm ($0.46d$) with $N=1$, $N=2$, $N=3$, geocells with $Z=0.25d$, $Z=0.5d$, $Z=0.75d$ & Cyclic plate load tests with same size of geocells and depth of placement with $N=1$ to $N=4$.

- Monotonic plate load tests on Geocell (made from geogrid CE-121) reinforced sand bed having $H_g=80$ mm ($0.61d$), $D_g=100$ mm ($0.78d$) with $N=1$, $N=2$, $N=3$, geocells with $Z=0.25d$, $Z=0.5d$, $Z=0.75d$ & Cyclic plate load tests with same size of geocells and depth of placement with $N=1$ to $N=4$.

- Monotonic plate load tests on Geocell (made from geogrid CE-121) reinforced sand bed having $H_g=130$ mm ($d$), $D_g=35$ mm ($0.27d$) with $N=1$, $N=2$, $N=3$, geocells with $Z=0.25d$, $Z=0.5d$, $Z=0.75d$ & Cyclic plate load tests with same size of geocells and depth of placement with $N=1$ to $N=4$. 

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- Monotonic plate load tests on Geocell (made from geogrid CE-121) reinforced sand bed having \( H_g = 130 \text{ mm} \) (d), \( D_g = 60 \text{ mm} \) (0.46d) with \( N=1, N=2, N=3 \), geocells with \( Z=0.25d, Z=0.5d, Z=0.75d \) & Cyclic plate load tests with same size of geocells and depth of placement with \( N=1 \) to \( N=4 \).

- Monotonic plate load tests on Geocell (made from geogrid CE-121) reinforced sand bed having \( H_g = 130 \text{ mm} \) (d), \( D_g = 100 \text{ mm} \) (0.78d) with \( N=1, N=2, N=3 \), geocells with \( Z=0.25d, Z=0.5d, Z=0.75d \) & Cyclic plate load tests with same size of geocells and depth of placement with \( N=1 \) to \( N=4 \).

- Monotonic plate load tests on Geocell (made from geogrid CE-121) reinforced sand bed having \( H_g = 180 \text{ mm} \) (1.38d), \( D_g = 35 \text{ mm} \) (0.27d) with \( N=1, N=2, N=3 \), geocells with \( Z=0.25d, Z=0.5d, Z=0.75d \) & Cyclic plate load tests with same size of geocells and depth of placement with \( N=1 \) to \( N=4 \).

- Monotonic plate load tests on Geocell (made from geogrid CE-121) reinforced sand bed having \( H_g = 180 \text{ mm} \) (1.38d), \( D_g = 60 \text{ mm} \) (0.46d) with \( N=1, N=2, N=3 \), geocells with \( Z=0.25d, Z=0.5d, Z=0.75d \) & Cyclic plate load tests with same size of geocells and depth of placement with \( N=1 \) to \( N=4 \).

- Monotonic plate load tests on Geocell (made from geogrid CE-121) reinforced sand bed having \( H_g = 180 \text{ mm} \) (1.38d), \( D_g = 100 \text{ mm} \) (0.78d) with \( N=1, N=2, N=3 \), geocells with \( Z=0.25d, Z=0.5d, Z=0.75d \) & Cyclic plate load tests with same size of geocells and depth of placement with \( N=1 \) to \( N=4 \).